The Criterions for Selecting Reference Points in RTK GPS Survey

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Key words:

SUMMARY

RTK GPS has seen great technological advances over the past few years and is now routinely used in a wide variety of engineering type applications. Around the world, real-time kinematics GPS systems provide with centimeter level position in seconds, allowing workers to get the precision they need at the moment that they need it. The one GPS receiver is placed in a control point (named the reference station) with known coordinates. RTK system combine GPS information and data radio communication with advanced algorithms to calculate precise position of large projects, such as highway and bridge project.

But the reference points are most important for surveying and the other applications. The accuracy of the reference points are acquired for survey and stake-out applications. In this article, we tested the most suitable positions for the reference points. And examining the reference points are how influence on the other points in this project.

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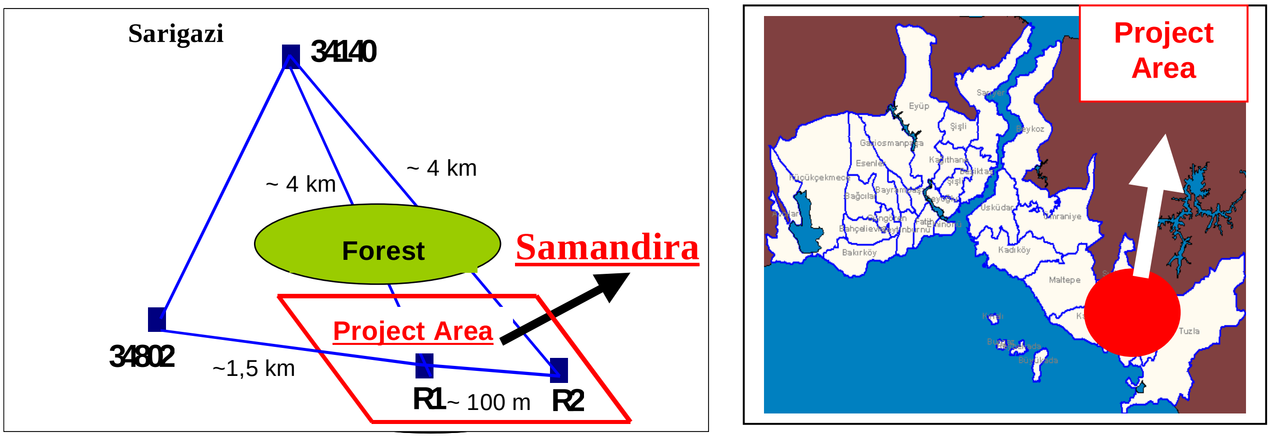
1. INTRODUCTION

RTK GPS, the one GPS receiver is placed in a control point (named the reference station) with known coordinates. By means of a radio connection between the reference station and the rover it is possible to transmit data from the reference station to the rover. The rover is placed in the point with unknown coordinates. The received data from the reference station are used for processing the by placing the basic points it has been taken into consideration that the basic points will be suitable for GPS reference stations for RTK measurements. It means that obstructions must not disturb the receiving of data from the satellites, and the location will ensure a stable radio communication between the reference station and the rover (no buildings, no tall dense trees) coordinates for the new point, immediately. Unlike using total stations for surveying the use of RTK means that it is not possible to measure all detailed points directly (e.g. corners of houses), because GPS requires free sight to the satellites /1/, /2/.

GPS survey was conducted in Samandıra region of İstanbul. The GPS data were acquired using three Ashtech GPS receivers. Data processing and network adjustments were conducted using Win Prism (Version 2.04).



Marmara See

Figure 1.

As you seen Figure 1, 34140 point is İstanbul GPS Triangulation Network Point which has to be connected with Turkey National Fundamental GPS Network TUTGA, by using GPS technique. This point has ITRF coordinate (survey one hour by GPS) and leveling measurements are being processed, for 1 km back and fore leveling range, the achieved root mean square error is ± 3.5 mm / km. 34140 point is fundamental leveling network point /5/.

Station 34140 (Figure 1) was chosen as the fixed station for network adjustment. Station 34140, R1, R2, P3, 9989, P118, P116, P4, 34802 have been derived from GPS data collected over 60 minute sessions using a measurement interval of 10 seconds. All baselines have ionosphere free, fixed integer solutions are all satisfactory. Station R1 and R2 are reference points in the project area.

The leveling measurements, one Topcon DL 102 digital leveling instruments and one barcode rods, which are made of fiberglass, have been used. By using these instruments, approximately 28 km leveling line is completed in 7 days.

While the leveling measurements are being processed for 1 km back and fore leveling range, the achieved root mean square error value is ± 2.5 mm / km /5/.

Table 1. Heights and Coordinates of 34140, R1 and R2 and the other reference points.

NN	φ_{ITRF}	Standard Deviation	λ_{ITRF}	Standard Deviation	H	Standard Deviation
34140	41° 0' 20''.64798		29° 12' 10''.30909		194.994	
R1	40° 58' 6''.94716	0.007	29° 12' 58''.91885	0.005	205.802	0.005
R2	40° 58' 4''.75308	0.007	29° 13' 1''.46492	0.005	207.695	0.005
P3	40° 57' 51''.95555	0.002	29° 13' 11''.76657	0.001	206.275	0.002
9989	40° 57' 20''.53529	0.005	29° 13' 3''.42920	0.003	298.501	0.004
P118	40° 58' 24''.21088	0.002	29° 12' 53''.20173	0.001	173.724	0.002
P116	40° 58' 13''.68333	0.005	29° 13' 0''.49586	0.003	191.617	0.004
P4	40° 58' 1''.93981	0.005	29° 13' 7''.06130	0.003	203.585	0.004
34802	40° 58' 26''.45751	0.002	29° 11' 39''.03802	0.002	271.665	0.002

2. TESTING USING RTK GPS BY DİFFERENT REFENCE POINTS

The reference point is very important for RTK GPS system and the accuracy results. We analyse where is the reference point is suitable for RTK GPS system. We performed this project in Samandıra Junction and are tested the getting results by the different reference points.

The approximately 20 points on the refuge are surveyed by means of the different reference points using RTK GPS system. Firstly, the accuracy of the reference points is depend on the transformation parameters.

Because of the large distortions in the conventional terrestrial geodetic networks, the transformation between such networks and GPS network is not easily achieved. In such kind of applications, instead of the mathematical transformation procedure, different transformation algorithms such as

In order to look at the scale differences between the ITRF and IGNA networks, a 3D transformation using Helmert model has been applied. In the transformation 4 common points are used. The results of the transformation are given in following Table 2.

<table>
<tr><th>Parameters</th><th>PN</th><th>Y</th><th>X</th><th>H</th></tr>
<tr><td rowspan="8">$T_X = 160^{g}.0832$
$T_Y = 68.7554$
$T_Z = 15.6346$
$W_X = 0.00099$
$W_Y = - 0.0006$
$W_Z = 0.0005$
Scale = 0.9999951
$m_0 = 0.0117$
$m_P = 0.0204$</td><td>R1</td><td>434071.942</td><td>4537567.400</td><td>169,055</td></tr>
<tr><td>R2</td><td>434130.869</td><td>4537499.183</td><td>170,947</td></tr>
<tr><td>P3</td><td>434365.243</td><td>4537181.270</td><td>169.527</td></tr>
<tr><td>9989</td><td>434161.606</td><td>4536213.712</td><td>261.766</td></tr>
<tr><td>P118</td><td>433940.038</td><td>4538180.220</td><td>136.972</td></tr>
<tr><td>P116</td><td>434107.680</td><td>4537853.920</td><td>154.866</td></tr>
<tr><td>P4</td><td>434257.962</td><td>4537490.263</td><td>166.836</td></tr>
<tr><td colspan="4">* This transformation is performed 4 points</td></tr>
</table>

Table 2. Transformation with helmert model

The results show that the translations, rotations and residuals are not significant. However the scale factor is statistically significant. Because of the large distortions in the conventional terrestrial geodetic networks, the transformation between such networks and GPS networks is not easily achieved. In such kind of applications, instead of the mathematical transformation procedure, different transformation algorithms such as interpolations and finite element methods are applied. The main goal is to find proper transformation equations fitting the structure of the network. The accuracy of the transformation depends on the size of the area, the accuracy of the terrestrial geodetic network points, the number and distribution of the common points and the developed model. In this study, such an algorithm enabling us to perform a reliable and easy use to transformation between ITRF and ED50. Then a transformation approach based on the multi parameter polynomial solution has been implemented /5/.

Using the ITRF94 and ED50 coordinates of each point, the differences between the geographical coordinates are written as follows:

$$\Delta\varphi = \varphi_{ITRF94} - \varphi_{ED50} \tag{1}$$

$$\Delta\lambda = \lambda_{ITRF94} - \lambda_{ED50} \tag{2}$$

These latitude and longitude differences can be expressed in the polynomial form by:

$$\Delta\varphi = A_0 + A_1 X + A_2 Y + A_3 X^2 + A_4 XY + A_5 Y^2 + \dots\dots \tag{3}$$

$$\Delta\lambda = B_0 + B_1 X + B_2 Y + B_3 X^2 + B_4 XY + B_5 Y^2 + \dots\dots \tag{4}$$

Considering $\Delta\varphi$ and $\Delta\lambda$ differences as the measurement values, the unknown A_i and B_i parameters can be estimated through the least squares adjustment, through this procedure the following final transformation equations have been obtained.

$$X = 1{,}7453293 * (\varphi_{ITRF} - 40^0) \quad (5)$$

$$Y = 1{,}7453293 * (\varphi_{ITRF} - 28^0) \quad (6)$$

$$\Delta\varphi = -161{,}992 + 60{,}4659.X + 3{,}0632.X^2 - 0{,}8938.X.Y \quad (7)$$

$$\Delta\lambda = -168{,}214 + 40{,}1292.Y + 0{,}2501.X.Y + 2{,}5697.Y^2 \quad (8)$$

$$\varphi_{ED50} = \varphi_{ITRF94} + 3''.4 - \Delta\varphi/1000 \quad (9)$$

$$\lambda_{ED50} = \lambda_{ITRF94} + 1''.5 - \Delta\lambda/1000 \quad (10)$$

The results show that the ITRF94 and ED50 datum are consistent with an accuracy of ± 5,3 cm along the meridian and ± 9,6 cm along the parallel directions. Using this algorithm, the ITRF 94 coordinates of main and densification network points have been transformed into ED50 datum. In our project, we used the two transformation models and the coordinates of the reference points were calculated /5/.

3. MODELING LOCAL GEOID

The geoid undulations, which are calculated from the below equation using the data on geoid base points, are visualized with the help of a “numerical terrain model”.

$$H = h - N \quad (11)$$

By this visualization, it is understood that the “Istanbul GPS/Leveling Geoid” has a regular trend. Geoid undulations, calculated from the GPS and leveling measurements, are modeled as two - parameter surface polynomial.

Here, a two – dimensional polynomial, which is in fifth order, is written and the variables of the polynomial, X and Y, are calculated via the equations (13, 14).

$$N = \sum_{i=0}^{5} \sum_{j=0}^{5} X^i Y^j C_{ij} \quad (12)$$

$$X = (B - 40^0) * 1{,}7453293 \quad (13)$$

$$Y = (L - 28^0) * 1{,}7453293 \quad (14)$$

$$N = A_{00} + A_{10}X + A_{01}Y + A_{20}X^2 + A_{11}XY + A_{02}Y^2 + A_{30}X^3 \ldots\ldots \quad (15)$$

$A_{00} = 36.8340$ $\quad$ $A_{10} = 1.8284$
$A_{01} = 0.64205$ $\quad$ $A_{20} = 1.6268$

In the equation (14, 15), B and L are the ellipsoidal latitude and longitude respectively.

The accuracy of the model is tested via independent leveling and GPS measurements on different parts of Istanbul Metropolitan area. It has been seen that the geoid undulations, which are calculated from the model, are conformed in geoid undulations, which are found from the GPS and leveling measurements, with in ± 5 cm. During the modelling process, the project area are considered and modeled separately. Then, the data for the project area is modeled as a compact geoid model. Heights for the reference points by using geoid model (Table 3) /5/.

PN	H_{ITRF}	N	$H=H_{ITRF}-N$	H(Leveling)
34140	194,994	36.771	158,223	158,223
R1	205,802	36.746	169,056	169,049
R2	207,695	36,7460	170,949	170,944
P3	206,275	36,7460	169,529	
9989	298,501	36,7360	261,765	261,680
P118	173,724	36,7485	136,976	
P116	191,617	36,7480	154,869	
P4	203,585	36,7470	166,838	

Table 3.

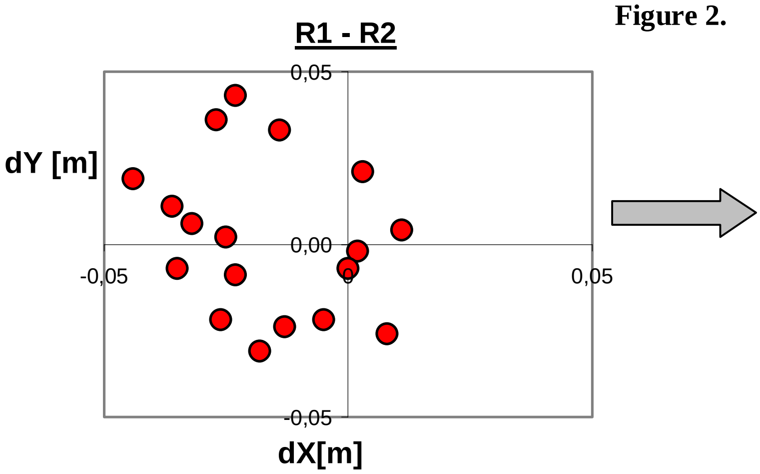
It is necessary to measure zenith angles with as high an accuracy as possible. We used Total Station (Geodimeter 520 N) and measured zenith angles (two series). Heights for the reference points by using Total Station (with using R1 Height).

Zenith angles are important for determining elevations in cases when it is practically impossible or economically unfeasible to employ spirit leveling. Their importance has increased with the introduction of Total Station instruments and will increase even more when determining refraction. So far their use in accurate determination of elevation is limited, because they are affected by atmospheric conditions much more than the horizontal angles. The scatter of the repeatedly measured zenith angles usually increases with increasing length of the line of sight and with decreasing elevation of the observation point and the effect of anomalous refraction is largest with grazing lines /3/, /4/, /1/, /2/.

PN	H (m)
P116	154,878
P4	166,818
P3	169,504
P118	136,978
9989	261,702

Table 4.

Figure 2.



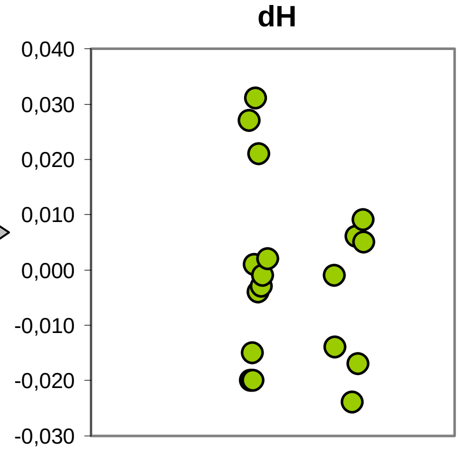


Figure 3

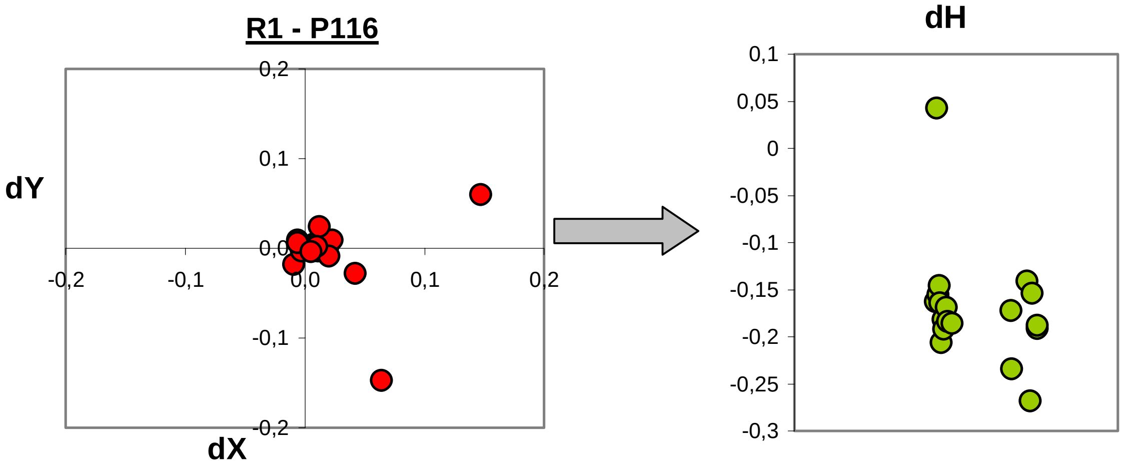


Figure 4.

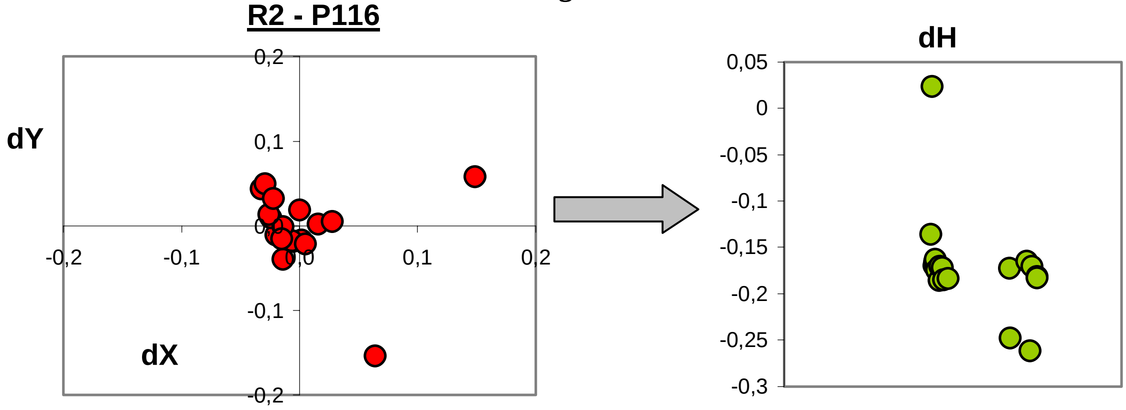


Figure 5.

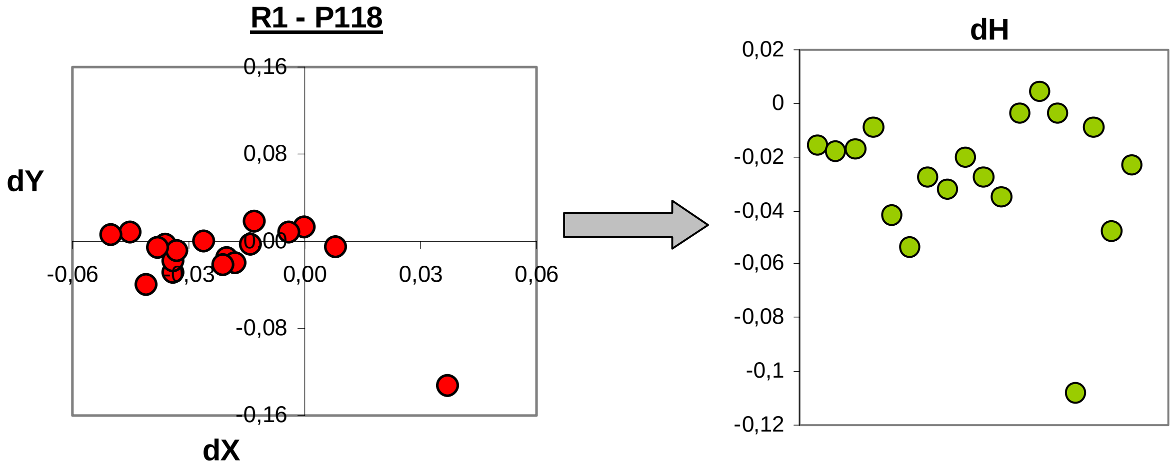


Figure 6.

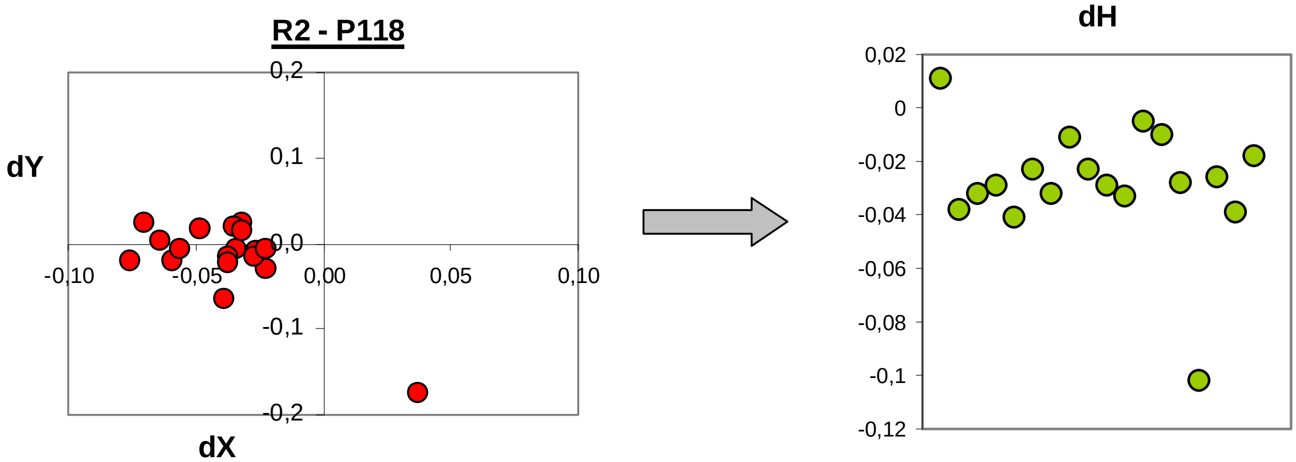


Figure 7.

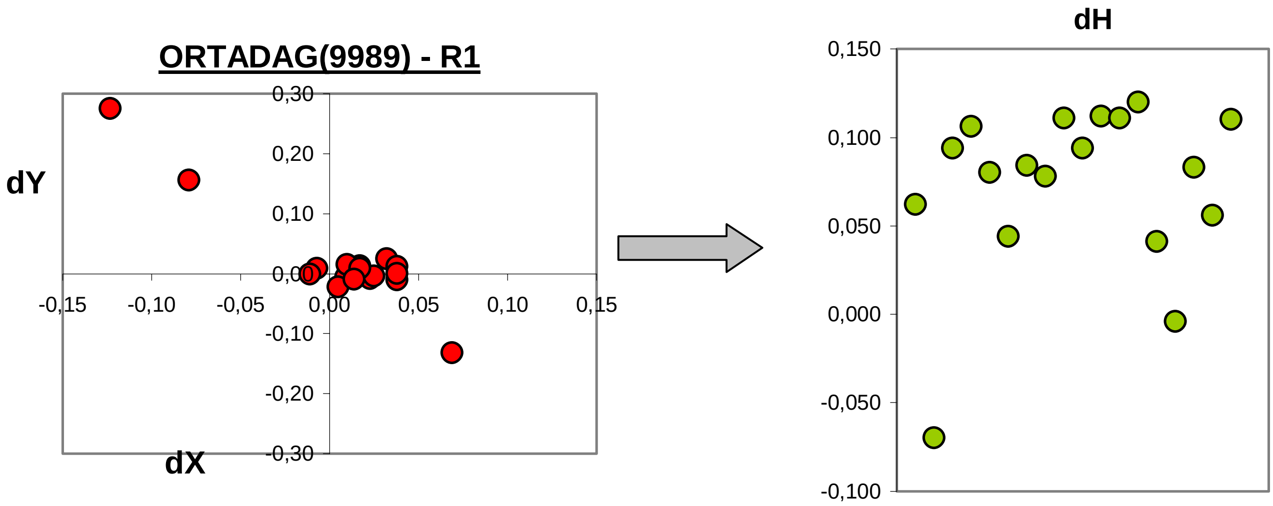


Figure 8.

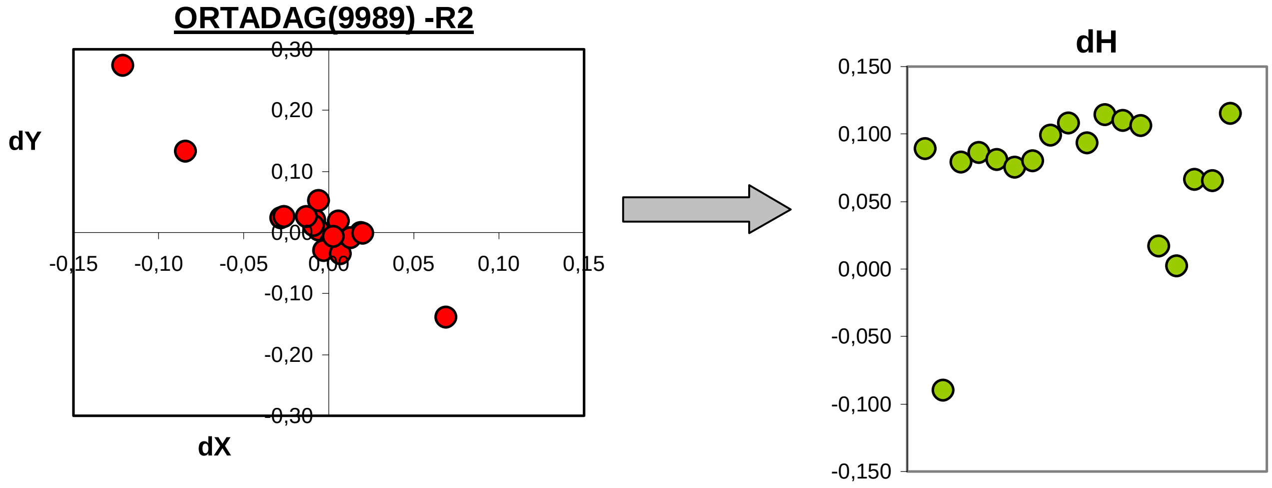


Figure 9.

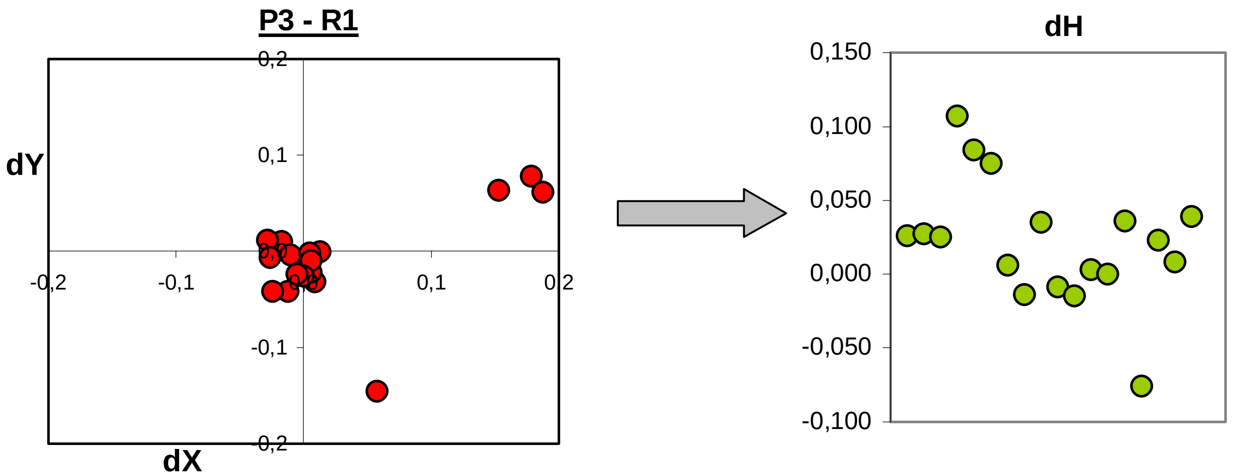


Figure 10.

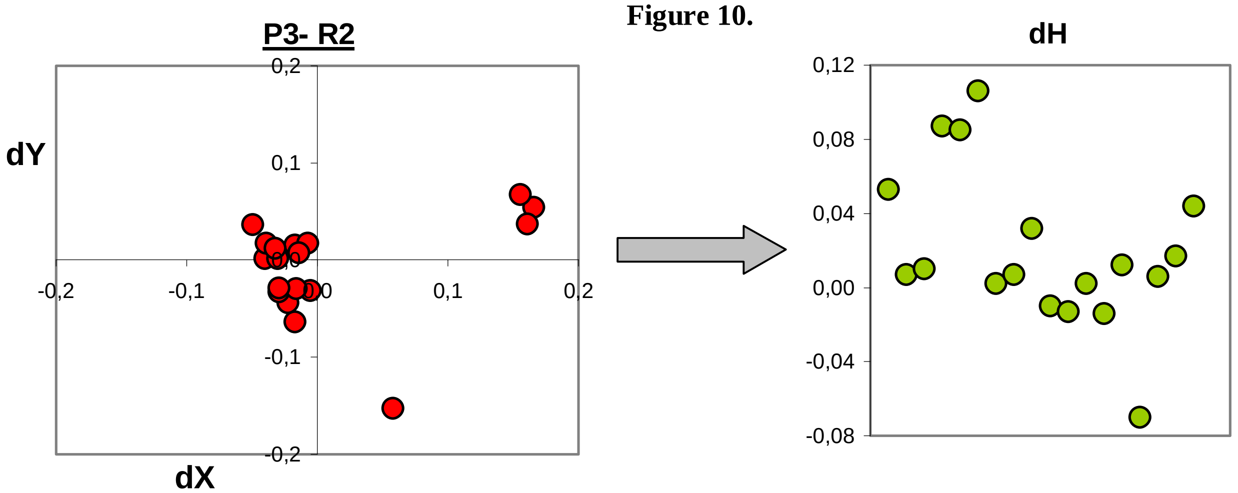


Figure 11.

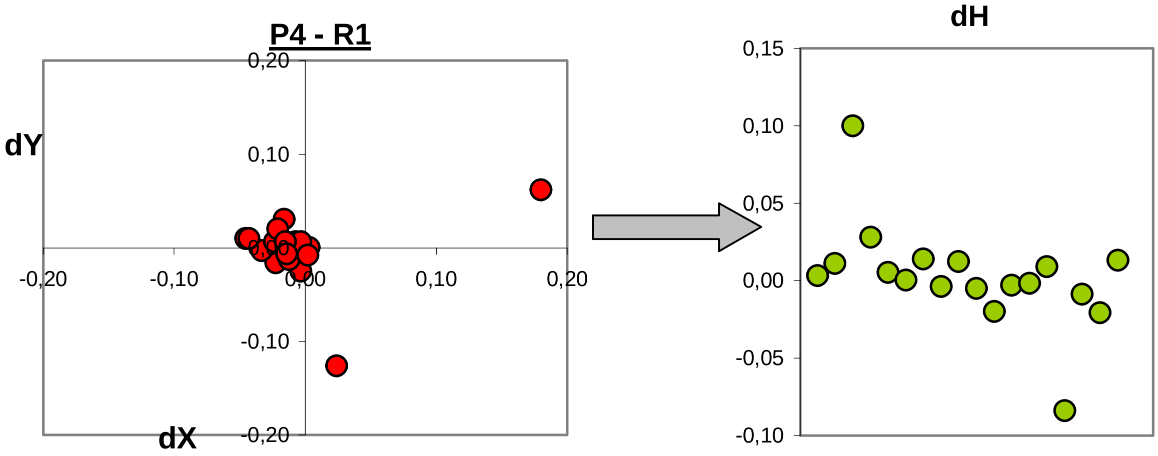


Figure 12.

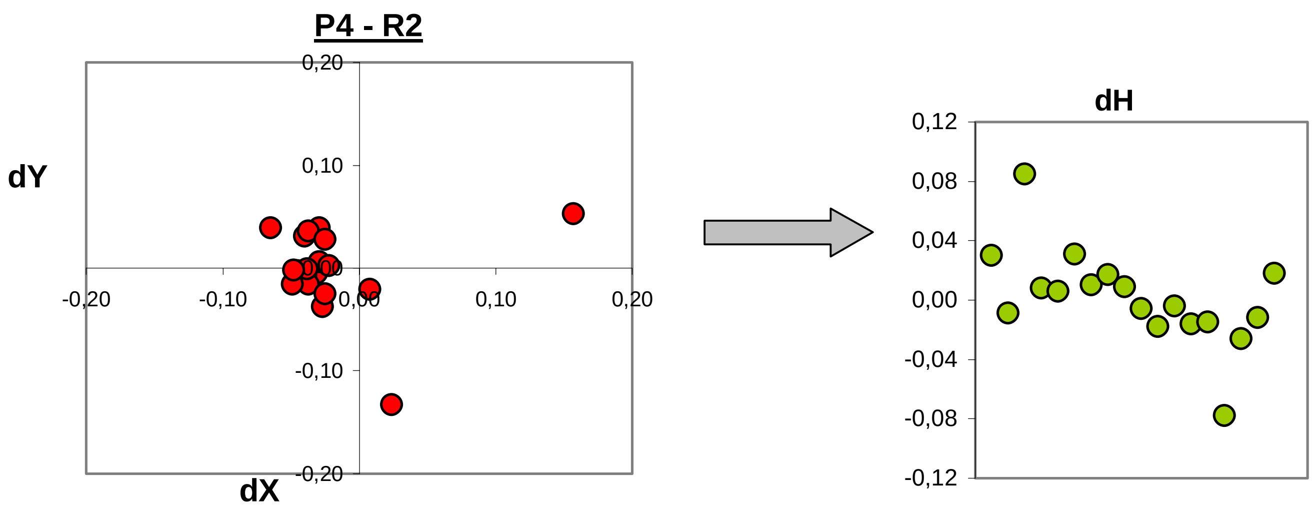


Figure 13.

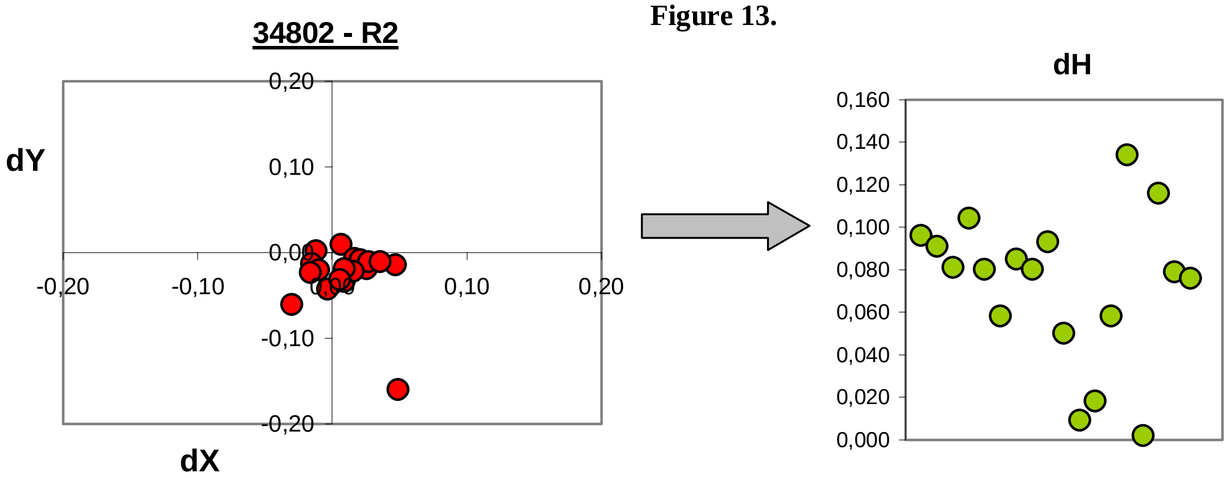
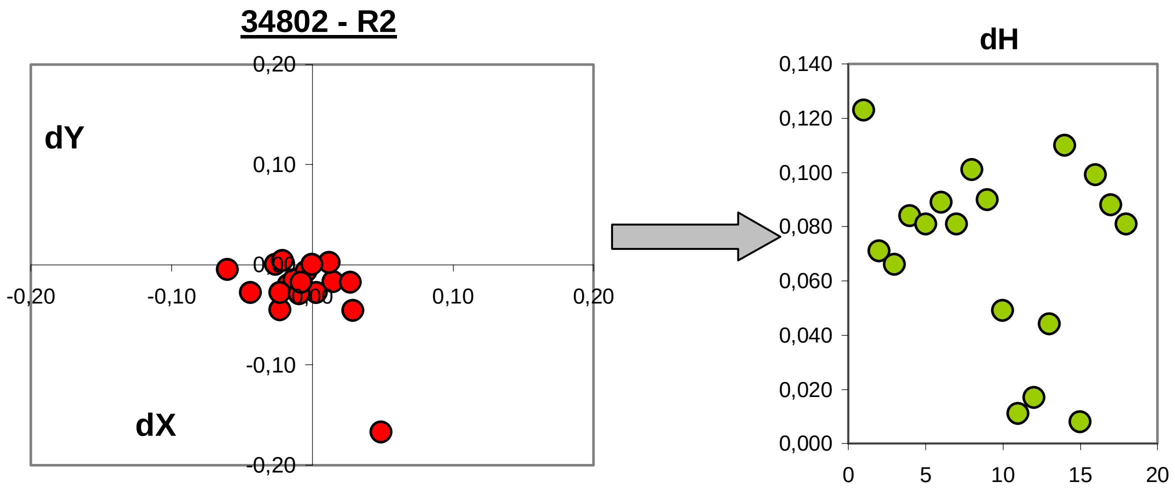


Figure 14.



As you see the above figures, especially for Figure 2 all of the results (the coordinate differences) are approximate 5 cm. However, the other figures results are 20 cm. In addition to 9989 reference point was located under the high voltage line. The elevation of this point having difference from the elevation which is obtained from leveling and trigonometric leveling. We suggest that the point located in the high voltage line, we should survey the elevation of this point by means of trigonometric or geometric leveling.

PN	Explanation
34140	*Basic triangulation point*
R1	*Reference point 1*
R2	*Reference point 2*
P3	*Near the highway*
9989	*Under the high voltage power line*
P118	*Very close to toll colection building*
P116	*Very close to forest area*
P4	*Near the highway*

Table 5.

The site selection of the reference point is a key element in reducing the time for planning GPS Surveys and post processing and analyzing GPS obsevational data. The following guideliness will be used for determinig the optimal location of newly established survey monuments /2/.

- At normal antenna height, clear view of the horizon above 15 degrees for 360 degrees in azimuth.
- Located on propert not likely to be disturbed.
- Located within 60 m of a parked vehicle
- Located on stable ground
- Readily accesible by vehicle
- Avoided tall artificial and natural reflective structure or surfaces that would project above the antenna ground plane

- Anticipated future road construction and tree growth
- Provided adequate site distances for conventional survey methods
- Adhered to airport requirements
- Ensured safety of surveyors and others
- Avoid radio towers and high transmission lines
- Located off road and within Highway right of way or other public property
- In the event power lines are unavoidable, mark should be placed opposite power lines such that lines are below 15 degrees above the horizon.

In this experiment, R1 and R2 was located in the middle and the heighest point of the project area. The results of the other points have not enough accuracy to performing RTK GPS survey.

4. CONCLUSION AND SUGGESTION

GPS measurement accuracy can be limited by ambiguity resolution, quality of ephemeris and starting coordinates, multipath, troposphere, antenna modelling and antenna height measurement. The effect of availability and quality of geoid models was also addressed.

This paper examined kinematic GPS results for surveying in using by different reference points. 9989 point is located under the high voltage line. The elevation of this point having difference from the elevation which is obtained from leveling and trigonometric leveling. We suggest that the point located in the high voltage line, we should survey the elevation of this point by means of trigonometric or geometric leveling.

As another results, it is aimed to use the products of this project for selecting the reference points in the middle and at the heighest point of the project area. These specifications of the reference points are neccessary for obtainig accuracy results in kinematic GPS survey. The important values of the reference points in GPS survey are C/NO, Elevation and Time graphics.

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